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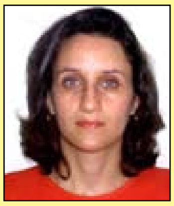
Resistant Core Section of High Strength Concrete Columns with Fibres Addition

Núcleo Resistente em Pilares de Concreto de Alta Resistência com Adição de Fibras Metálicas




A.E.P.G.A. JACINTHO [a]
paganell@fec.unicamp.br

J.S. GIONGO [b]
jsgiongo@eesc.usp.br

T.N. BITTENCOURT [c]
tbitten@usp.br

A.C. ARAÚZ [d]
achavez_arauz@yahoo.com.br

Abstract

This experimental work investigation is about concrete columns of high strength silica fume and steel fibre reinforced concrete, under centric load, using concrete reinforcement cover of 2 cm and 5cm. Square cross section of columns of 20cm x 20cm and 120cm high was used for all specimens, but stirrup spacing and fibre reinforcement were varied. The longitudinal reinforcement ratio remained unchanged. The objective of this work was to analyze if increasing the concrete cover thickness of the models the cross section strenght remains the same. Other researches have proved that the resistant cross section is delimited only by the stirrups, for circular, square or rectangular section. In this work it was observed that increasing the cover thickness contributes to a resistant cross section of columns.

Keywords: columns, high strenght concrete, fibre concrete, steel fibres, cover, resistant cross section.

Resumo

Este trabalho visa analisar os resultados experimentais obtidos após estudo do comportamento de pilares, moldados com concreto armado de alta resistência com adição de sílica ativa e fibras metálicas, submetidos à compressão simples, com 2cm de espessura de cobrimento da armadura e com 5cm de espessura. A seção transversal dos pilares era quadrada, de 20cmx20cm e altura de 120cm, para todos os modelos, com espaçamento entre estribos e taxa volumétrica de adição de fibras metálicas variados. A taxa de armadura longitudinal permaneceu a mesma para todos os modelos. O objetivo deste trabalho foi a verificação se mesmo com o aumento da espessura do cobrimento da armadura, o mesmo não influenciaria a seção resistente do pilar, visto que em outras pesquisas, ficou comprovado que esta seção é formada apenas pelo núcleo da seção transversal, tanto para pilares de seção circular quanto para elementos de seção transversal quadrada ou retangular. Como resultado observou-se que a medida que se aumenta a espessura do cobrimento, este passa a contribuir com a seção resistente do pilar para a ação da força última.

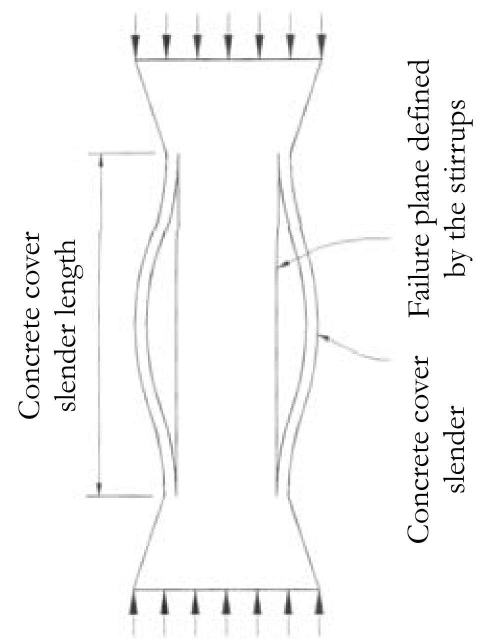
Palavras-chave: pilares, concreto de alta resistência, concreto com fibras, fibras metálicas, cobrimento, seção resistente.

a – UNICAMP-FEC-DES paganell@fec.unicamp.br Caixa Postal 6021 CEP 13083-852 – Campinas – S.P. - Brasil
b – USP-EESC-SET jsgiongo@eesc.usp.br Av. Trabalhador São Carlense, 400 CEP13566-590 – São Carlos – S.P. – Brasil
c – USP-POLI-PEF tbitten@usp.br Av. Prof. Almeida Prado tv.2 n.271 CEP 05580-900 – São Paulo – S.P. – Brasil
d – UNICAMP-FEC-DES achavez_arauz@yahoo.com.br Caixa Postal 6021 CEP 13083-852 – Campinas – S.P. – Brasil

1 Introduction

The behaviour of HSC (high strength concrete) columns, according to LANGLOIS (9), is characterized by the fast and sudden rupture of the concrete cover. Contributing to this phenomenon is the embrittlement in the interface between the confined concrete and not confined concrete (cover), created by the reinforcement. In GUIMARAES (7), it was concluded that the fibres avoid the premature separation of the concrete cover in the columns, therefore the effect of the randomly located fibres in the concrete mass, delays this rupture before the columns reach the collapse. The premature detachment of the concrete cover in the columns with HSC is observed when concrete with superior strength – 80 MPa or more – are used. HELENE (8) discusses the durability of the concrete structures in relation to the normally used thickness for the concrete cover of the longitudinal bars. According to him, the cover is a limitant factor of the durability of the structures, but it implies in an increase of the building cost.
Figure 1 illustrates the buckling of the concrete plate that constitutes the concrete cover when the columns are carried with the load, which is reported in (9). With a minimum of fibre reinforcement of the concrete, the buckling doesn't take place anymore , since the fibres "sew" the cover to the core, but without making the total cross section more resistant to the applied load in the column.

Figure 1 – Detachment of the Reinforcement Cover (1)



High strength concrete with fibre reinforcement (HSFC) is mainly used for floors, to increase fatigue and bending resistance, thus as well as the durability and to decrease the openings of the cracks. This quality of fibres in cracking control can be used to avoid the premature detachment of the concrete cover of loaded columns, while increasing the ductility, which is a desired quality in structural projects submitted to seismic action. It's also used in structures where placing stirrups is impossible or when the preparation and assembly of these structures represent a considerable costs.
The works developed in Canada focus mainly on the strenght of concrete to applications of cyclical loading, as reported in (9) and LEVESQUE (10), where studies were made about the applicability of concrete with steel fibres in columns under cyclical centered load to increase the confinement effect of the core of the elements, given by stirrups, and the ductility under the action of cyclical loading.
The researchers concluded that the fibres increase the confinement effect, since this effect is obtained mainly by the stirrup arrangement.
The research developed in this work had its beginning in a doctoral thesis and continued during the post-doctoral course at the Structure Laboratory of EESC (the Engineering School of São Carlos) and the Structure and Structural Material Laboratory of the EP, both from the University of São Paulo.

2 HSC Study in Columns

The subject high strength concrete has been highly discussed in meetings and congresses throughout the world, because of its great employability in high-rise buildings due to the reduction of the dimensions of columns built with this material. But there are still many doubts in regard to its use, arousing therefore the researcher's interest. The purpose of this work is to give technical support for a safe use of this concrete in building construction.
In this work where a part of the research is reported with more details in (7) the tests were carried out on columns submitted to centric compression with load control. The behaviour of the columns could be studied through the Stress x Strain diagrams in the ascending phase of the curve until the rupture of the elements. Dimensions of the models can be seen in Figure 2.
The test control through load control enables the characterization of the collapse when the ultimate strength is reached, because the piston of the equipment searches for stiffness as response to its pressure. When the model reaches the ultimate strenght, it loses stiffness suddenly, making the piston find this stiffness in the model's support. This effect does not occur when the test control is done through displacement control, therefore the equipment does not search for stiffness anymore as response to the applied pressure, but for the corresponding displacement imposed by the piston of the equipment. With this kind of test control the element behaviour can

be observed after its rupture through the observation of the stress redistribution capacity and its ductility .
The first stage of this work consists of tests on columns with a reinforcement cover thickness of 2cm, tested under centric compression with load control and the second stage consists of tests of 9 columns with a 5cm reinforcement cover, tested under centric compression with displacement control.
BJERKELI et al.(4) present as part of a program that was developed in Norway, a ductility study of concrete columns with a normal specific mass and strength of 65MPa to 115MPa, and light concrete from 60MPa to 90MPa, with strength measured through cubic specimens with 10cm edge. According to the authors the factors that influence the structural behaviour are: the geometric configuration, distribution and amount of longitudinal reinforcement, concrete strength, aggregate type and loading speed. Therefore it's important to develop solutions that present a reinforcement reduction and the required ductility.
For stirrup reinforcement rates of 1.1% and 3.1%, a ductility variation was observed when the diameter of the longitudinal bars were increased from 10mm to 16mm, a square transversal section of 150mm and 500mm effective height. In this series the aim was to obtain information about the geometry of the transversal section. The other parameters were the amount and distribution of the confinement reinforcement, the influence of the longitudinal reinforcement and loading speed. Also circular cross-section columns were tested.
The (4) research verified that a more ductile behaviour occurs in the circular section columns with spiral transversal reinforcement, although these columns did not have any longitudinal reinforcement. Rectangular cross section columns with a higher number of longitudinal reinforcement bars and transversal reinforcement formed by stirrups and horizontal mesh bars, presented a little less ductile behaviour, but better than the square cross section columns. But it must be observed that in these columns destined to confinement analysis and its influence in ductility, the concrete cover wasn't used, which is not common practice in civil construction.
CUSSON & PAULTRE (6), presented an experimental study about HSC columns, confined by rectangular stirrups. 27 columns were tested and the dimensions can be seen in figure 3, all of them with concrete cover. The compression strength of four models, after 28 days, was less than 90MPa and the others from 93.1MPa to 115.9MPa, determined with 15cm x 30cm cylindrical specimen. The transversal reinforcement yield stress, stirrup spacing and configuration, longitudinal and transversal reinforcement ratios were also variable.
In general it was observed that the behaviour was characterized by the sudden rupture of the concrete cover. This work was developed in the Civil Engineering Laboratory of the University of Sherbrooke (Sherbrooke, Quebec - Canada) which concluded that in axial compression just the concrete core area, delimited by the transversal reinforcement, must be considered to calculate the axial compression strength of HSC columns, because the behaviour of such columns presented a sudden failure of the concrete cover, so the cover does not participate in the strength capacity of the column (figure 1).
Considering the low confinement efficiency of HSC compared to normal strength concrete, an increase of strength and ductile behaviour of the HSC confined columns were obtained after adopting adequate longitudinal and transversal reinforcement detailing .

Figure 2 – Dimension of the Models

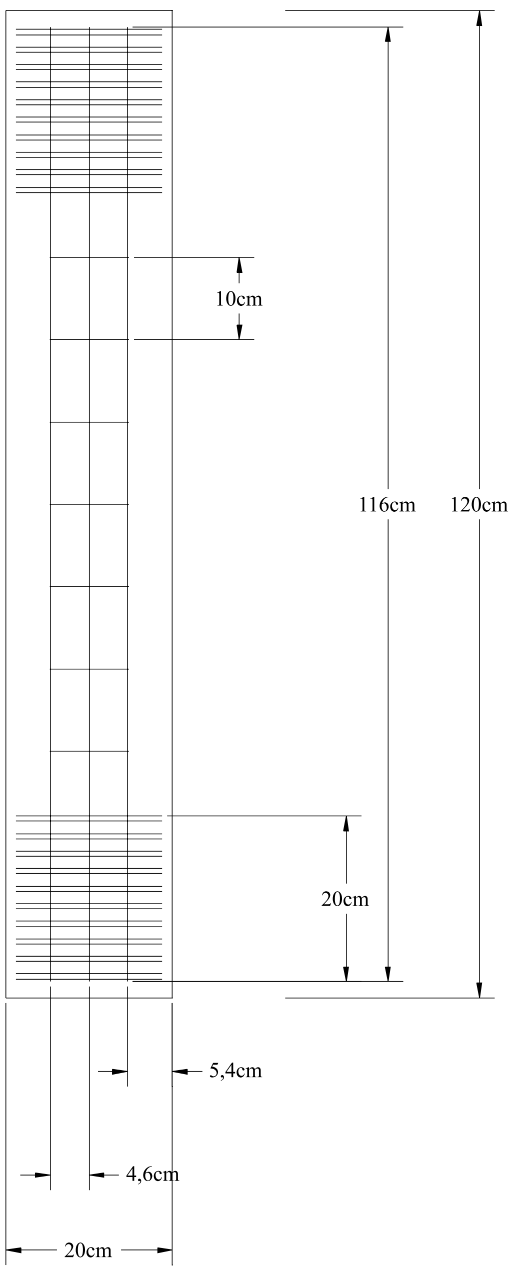
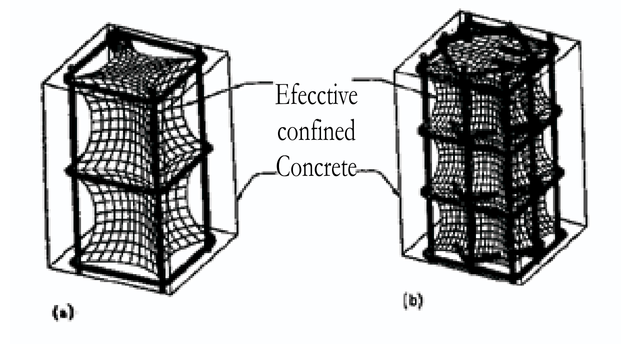


Figure 3 – Confinement Effect (6)



LIMA et al.(11) developed researches on high strength concrete columns, testing the models on centric and eccentric compression with load control applied on the models.

The columns tested under axial compression had 4 series differentiated by the size of the models and the spacing of stirrups. In two series, the cross section of the models was square, with 20cm x 20cm and 120cm height, while in the other two series the models had rectangular cross section with 15cm x 30cm and 90cm height.

One of the conclusions reached in this work was that there aren't great modifications in the behaviour under centric compression because of the change from square to rectangular cross section. Another conclusion was that the resistant cross section of the columns was formed by the core delimited by the stirrup axis, just as in (6) and (4).

3 Results of Columns Tests

In figures 4 to 15, the stress x strain diagrams tested in the first stage of this research can be seen. Two columns of each series were tested, totalizing 26 columns. The fibre reinforcement varied from 0.25% to 1.00%, in volumetric ratio, and the stirrup spacing was 05cm, 10cm and 15cm. All tests were executed in a space frame, whereas the load carried on the models were controlled. With the rupture of the models, the reading of the strain and the relative displacements were lost. The reading was carried out by strain gauges and comparing clocks attached to the columns.

Figure 16 to figure 24 show the Stress x Strain diagrams of the second stage of this research, where the columns were tested with displacement control, testing one column of each series, totalizing 9 columns where the volumetric ratios of the fibre reinforcement were 0.50%, 0.75% and 1.00%, and the stirrup spacing varied in the same way as seen in the previous sequence of tests: 05cm, 10cm and 15cm.

Except for the columns of the P40a05 and P40a15 series, all the other models were tested in a space frame, with displacement control, obtaining the Stress x Strain curve of the columns with the ascending and descending part. The test of the P40a05 column was carried out with the

Figure 4 – Stress x Strain Diagram of the P1a15 series

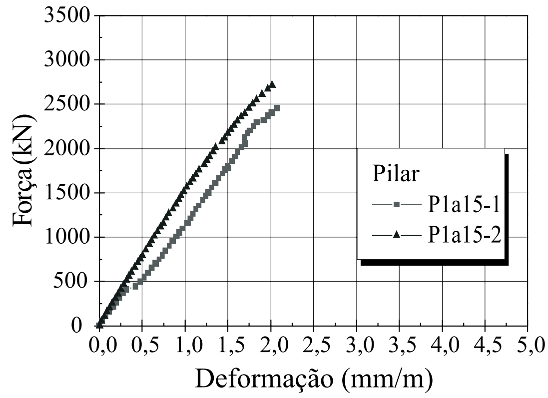


Figure 5 – Stress x Strain Diagram of the P1a10 series

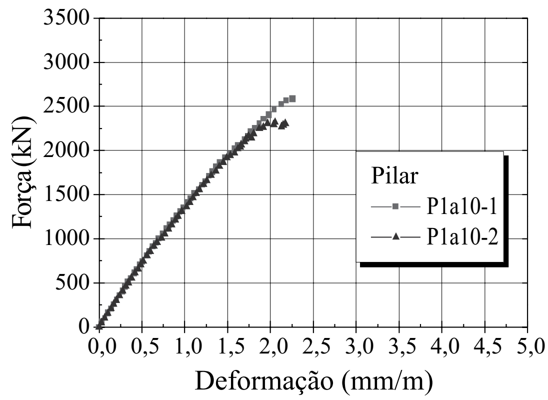


Figure 6 – Stress x Strain Diagram of the P1a05 series

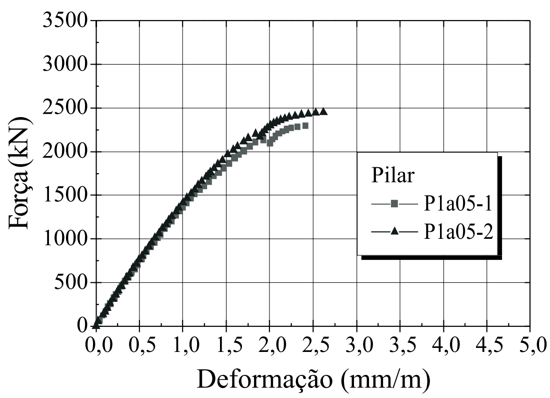


Figure 7 – Stress x Strain Diagram of the P2a15 series

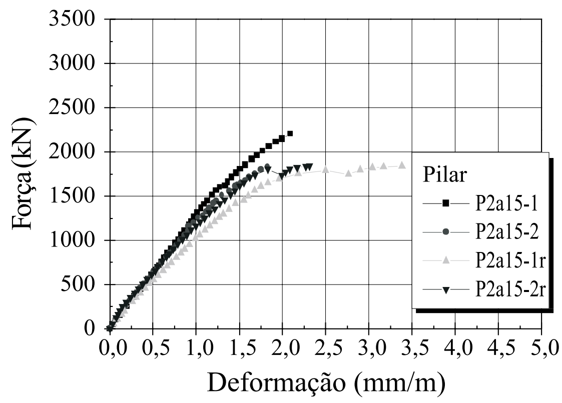


Figure 10 – Stress x Strain Diagram of the P3a15 series

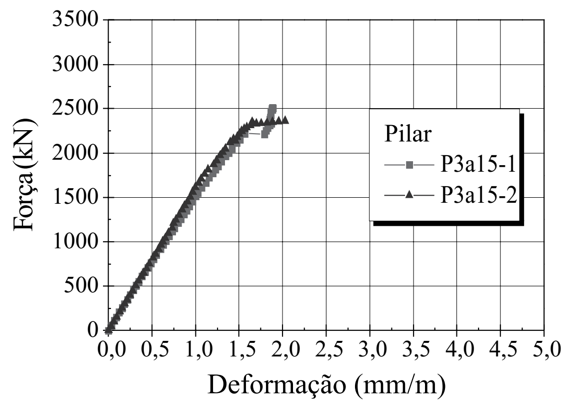


Figure 8 – Stress x Strain Diagram of the P2a10 series

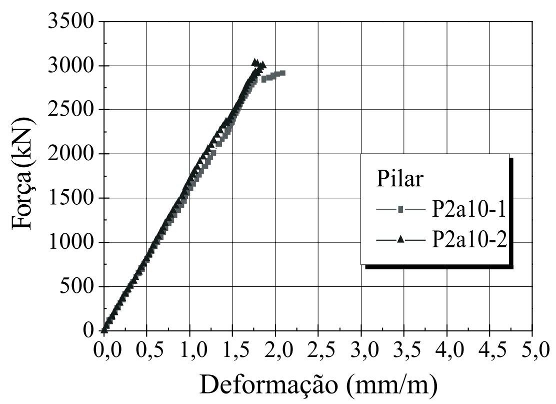


Figure 11 – Stress x Strain Diagram of the P3a10 series

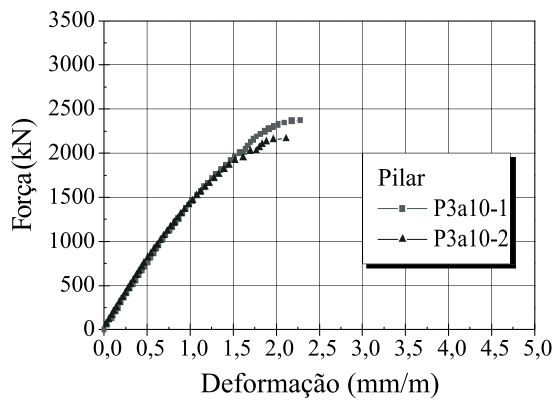


Figure 9 – Stress x Strain Diagram of the P2a15 series

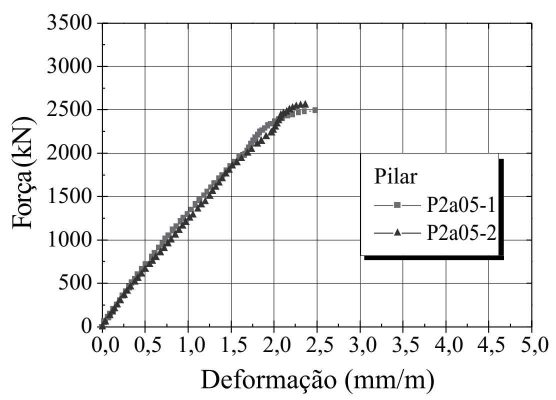


Figure 12 – Stress x Strain Diagram of theP3a05 series

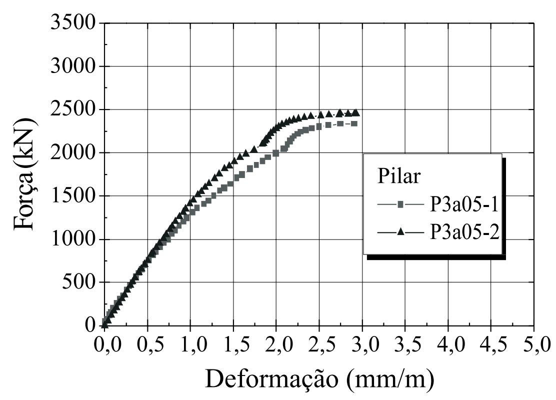


Figure 13 – Stress x Strain Diagram of the P4a15 series

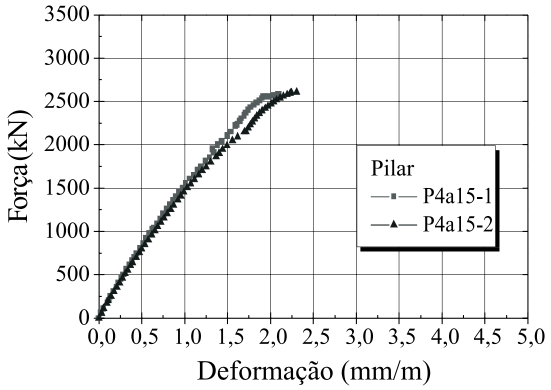


Figure 14 – Stress x Strain Diagram of the P4a10 series

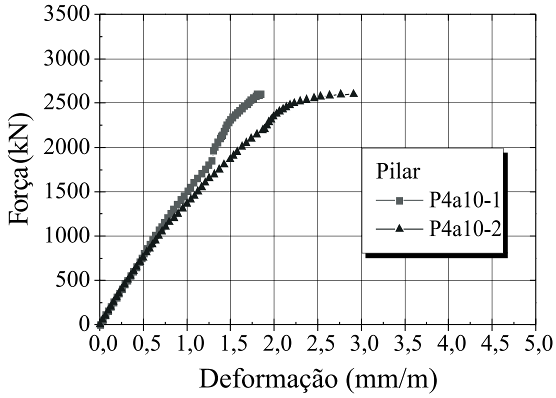
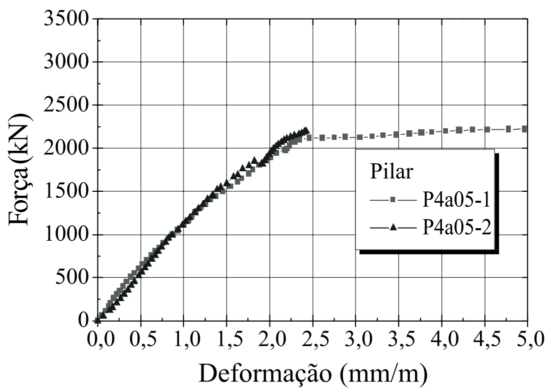


Figure 15 – Stress x Strain Diagram of the P4a05 series



INSTRON universal test machine with displacement control. The test of the P40a15 column was carried out with load control to reach the collapse of the column trying to get the descending part of the Stress x Strain curve with data acquisition equipment. After the rupture of the column the readings of the comparing clocks were lost. As before in the final test in the space frame with load control, this same column was tested with the INSTRON machine, the two obtained graphics were compared to verify if there was any significant loss of strength. It was observed that the column lost very little of its strength in one test compared to the other, thus validating the results obtained in the test with load control.

4 Analysis of Tests Results

To analyze the results, it's first necessary to obtain some parameters to compare the experimentally obtained results against the theoretical results.

Figure 16 – Stress x Strain Diagram of the P40a05 series

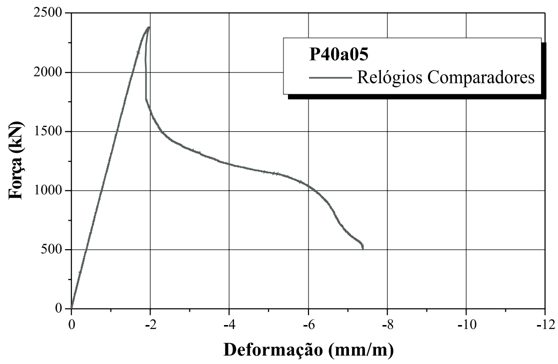


Figure 17 – Stress x Strain Diagram of the P40a10 series

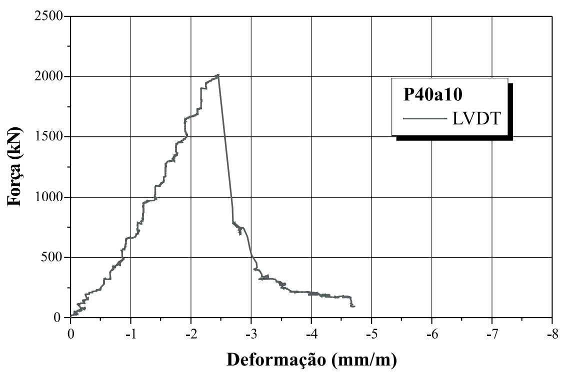


Figure 18 – Stress x Strain Diagram of the P40a15 series

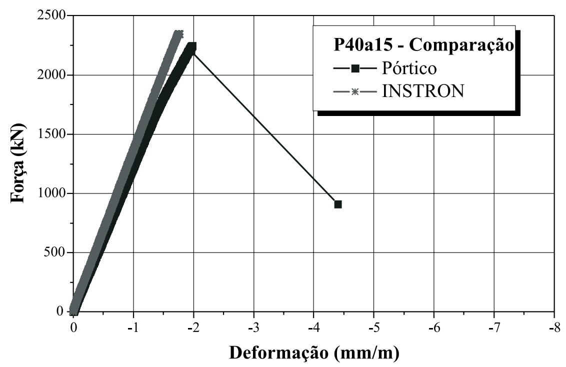


Figure 21 – Stress x Strain Diagram of the P60a15 series

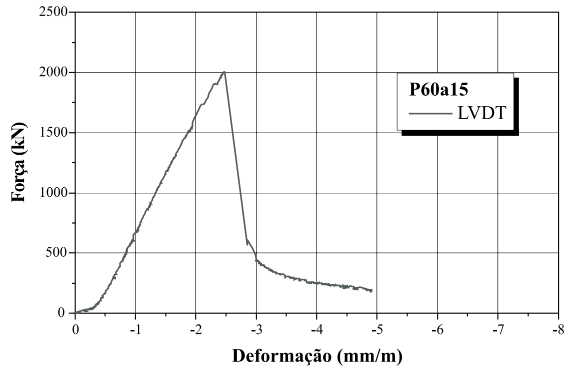


Figure 19 – Stress x Strain Diagram of the P60a05 series

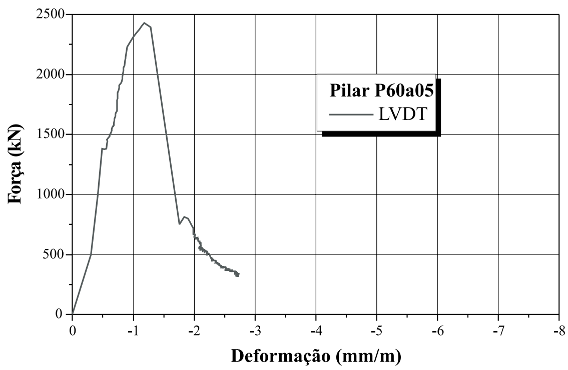


Figure 22 – Stress x Strain Diagram of the P80a05 series

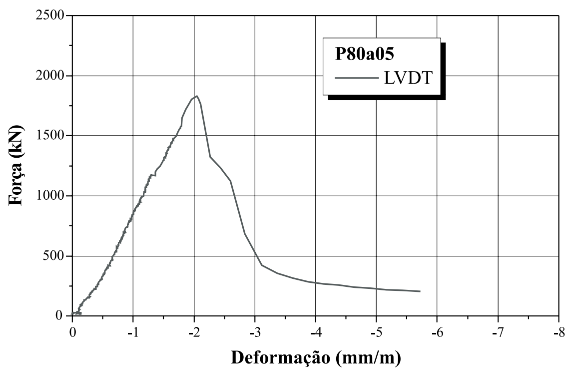


Figure 20 – Stress x Strain Diagram of the P60a10 series

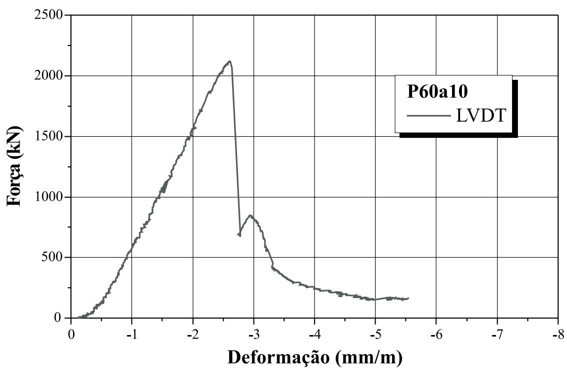


Figure 23 – Stress x Strain Diagram of the P80a10 series

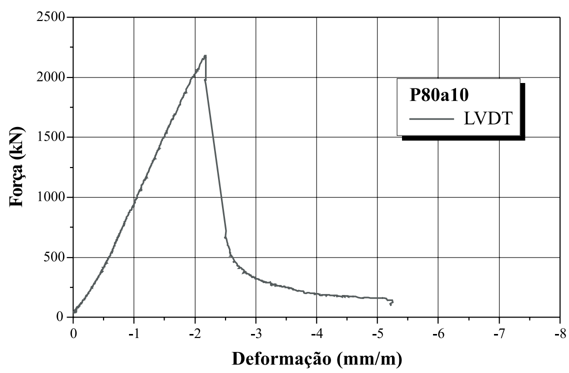
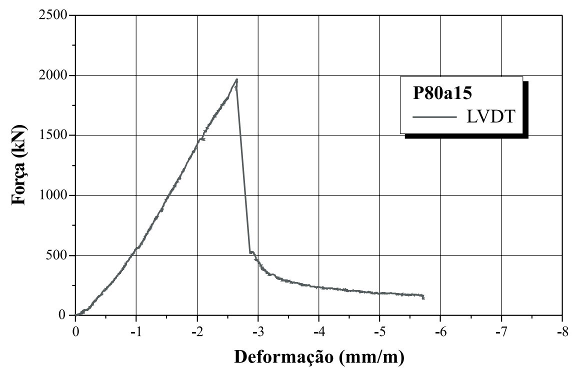


Figure 24 – Stress x Strain Diagram of the P80a15 series



For the transversal reinforcement ratio calculation, equation 1 was used as described below:

$$\rho_w = \frac{A_{sw}}{b_w\, s} \qquad \textbf{(Equation 1)}$$

where:

A_{sw} = stirrup area;
b_w = smallest dimension of the cross section;
s = stirrup spacing.

To calculate the ultimate strength (F_{un}) using the core cross section, equation 2 was used:

$$F_{un} = (A_{cn} - As)f_c + A_s f_y \qquad \textbf{(Equation 2)}$$

where:

A_{cn} = concrete core delimited by the stirrups;
A_s = longitudinal bar area;
f_c = average concrete strength;
f_y = average yield strength.

The ultimate strength (F_u) using the total cross section was calculated through equation 3:

$$F_u = (A_c - A_s)f_c + A_s f_y \qquad \textbf{(Equation 3)}$$

Table 1 shows the experimental and the theoretical results of the first stage of tests, the relation between the experimental strength (obtained in tests) and the theoretical strength calculated with equation 2 and equation 3.
It was observed that when the experimental results are compared with the theoretical strength calculated with the total cross section, the factors, in its majority lower than 1, indicated values against the safety.
When the core cross section is considered, the analysis provides experimental results above of the theoretical results.

This clearly means that for a concrete cover of 2cm, this part of the cross section does not contribute to the strength of the cross section of the columns.
Figure 25 shows the rupture of one of the tested columns. As seen in the picture there is no detachment of the cover before the rupture.
COLLINS et al.(5) proposed the K_3 coefficient multiplying with the resistant part of concrete to consider the total cross section of columns as described:

$$F_{u,teo} = K_3(A_c - A_s)f_c + A_s f_y \qquad \textbf{(Equation 4)}$$

$$K_3 = 0{,}6 + \frac{10}{f_c'} \quad \text{in MPa} \qquad \textbf{(Equation 5)}$$

The fibre reinforcement decreases the compression strength, but as the volumetric ratio of fibres was lower than indicated in

Figure 25 – General view of column failure tested in the first stage

technical literature, there wasn't any difference in the concrete strength because of the fibre reinforcement in both the first stage and the second stage tests. It's necessary to carry out a higher number of tests to indicate another coefficient to replace K_3 to calculate the strength, considering both the total cross section and the fibre reinforcement in the concrete.
Table 2 shows an analysis between values obtained for the theoretical strength, using the K_3 coefficient suggested by (5), with the experimental results. The equations indicated by (5) are based on the determination of the concrete compression strength determined by 15cm x 30cm cylindrical specimens. Since 10cm x 20cm cylindrical specimens were used in the tests to determine the average compression strength of concrete (f_{cm}), f_{cm} is multiplied by the 0,95 coefficient for adjustment to consider the difference of the dimensions of the specimens .

For the second stage tests, now with a higher concrete cover thickness, the following analyses were made. Table 3 presents the experimental results of columns, the ultimate theoretical strength considering the core delimited by stirrups, and the relation between the experimental and the theoretical results.
It can be noted that the relation between the experimental ultimate strength and the theoretical ultimate strength only considering the core of cross section had values higher than 1, indicating that in this case the cover may have contributed to the strength of the cross section.
In figure 26 a typical failure of the tested columns can be seen in the second stage.
Similarly the experimental results were compared with those calculated according to COLLINS et al.(5), as shown in table 2.

Table 1 – Theoretical Analysis of Tests Results

Column	Vf (%)	ρw (%)	$0,9_{fcm}$ (MPa)	$F_{u,exp}$ (kN)	$F_{u,teo}$ (kN)	$F_{u,exp}/F_{un,teo}$	$F_{un,teo}$ (kN)	$F_{u,exp}/F_{un,teo}$
P1a15-1	0,25	0,55	73	2.453	3.383	0,73	2.303	1,07
P1a15-2	0,25	0,55	79	2.714	3.621	0,75	2.451	1,11
P1a10-1	0,25	0,82	77	2.581	3.539	0,73	2.400	1,08
P1a10-2	0,25	0,82	77	2.304	3.539	0,65	2.400	0,96
P1a05-1	0,25	1,63	73	2.291	3.371	0,68	2.295	1,00
P1a05-2	0,25	1,63	73	2.449	3.371	0,73	2.295	1,07
P2a15-1	0,50	0,55	65	2.208	3.061	0,72	2.103	1,05
P2a15-2	0,50	0,55	65	1.827	3.061	0,60	2.103	0,87
P2a15-1r	0,50	0,55	60	1.840	2.871	0,64	1.985	0,93
P2a15-2r	0,50	0,55	60	1.841	2.871	0,64	1.985	0,93
P2a10-1	0,50	0,82	72	2.911	3.346	0,87	2.280	1,28
P2a10-2	0,50	0,82	72	3.028	3.346	0,91	2.280	1,33
P2a05-1	0,50	1,63	70	2.491	3.264	0,76	2.229	1,12
P2a05-2	0,50	1,63	70	2.554	3.264	0,78	2.229	1,15
P3a15-1	1,00	0,55	69	2.509	3.244	0,77	2.217	1,13
P3a15-2	1,00	0,55	69	2.360	3.244	0,73	2.217	1,06
P3a10-1	1,00	0,82	59	2.373	2.821	0,84	1.954	1,21
P3a10-2	1,00	0,82	59	2.164	2.821	0,77	1.954	1,11
P3a05-1	1,00	1,63	62	2.333	2.962	0,79	2.041	1,14
P3a05-2	1,00	1,63	62	2.454	2.962	0,83	2.041	1,20
P4a15-1	0,75	0,55	72	2.584	3.342	0,77	2.277	1,14
P4a15-2	0,75	0,55	72	2.609	3.342	0,78	2.277	1,15
P4a10-1	0,75	0,82	78	2.603	3.573	0,73	2.421	1,08
P4a10-2	0,75	0,82	78	2.598	3.573	0,73	2.421	1,07
P4a05-1	0,75	1,63	68	2.222	3.190	0,70	2.183	1,02
P4a05-2	0,75	1,63	68	2.199	3.190	0,69	2.183	1,01

V_f(%): volumetric fibres ratio added into the concrete;
ρ_w(%): volumetric steel transverse ratio;
$F_{u,exp}$: experimental ultimate strenght;
$F_{u,teo}$: theoretical ultimate strenght considering the total cross section of column;
$F_{un,teo}$: theoretical ultimate strenght considering the core cross section column;

Table 2 – Theoretical Analysis according COLLINS et al.(1993)

Column	Vf (%)	0,95$_{fcm}$ (MPa)	K_3	$F_{u,exp}$ (kN)	$F_{u,teo}$ (kN)	$F_{u,exp}/F_{u,teo}$
P1a15-1	0,25	77	0,730	2.453	2.730	0,90
P1a15-2	0,25	83	0,720	2.714	2.881	0,94
P1a10-1	0,25	81	0,723	2.581	2.828	0,91
P1a10-2	0,25	81	0,723	2.304	2.828	0,82
P1a05-1	0,25	77	0,730	2.291	2.720	0,84
P1a05-2	0,25	77	0,730	2.449	2.720	0,90
P2a15-1	0,50	68	0,747	2.208	2.527	0,87
P2a15-2	0,50	68	0,747	1.827	2.527	0,72
P2a15-1r	0,50	63	0,758	1.840	2.405	0,77
P2a15-2r	0,50	63	0,758	1.841	2.405	0,77
P2a10-1	0,50	76	0,732	2.911	2.707	1,08
P2a10-2	0,50	76	0,732	3.028	2.707	1,12
P2a05-1	0,50	74	0,736	2.491	2.655	0,94
P2a05-2	0,50	74	0,736	2.554	2.655	0,96
P3a15-1	1,00	73	0,737	2.509	2.643	0,95
P3a15-2	1,00	73	0,737	2.360	2.643	0,89
P3a10-1	1,00	62	0,762	2.373	2.374	1,00
P3a10-2	1,00	62	0,762	2.164	2.374	0,91
P3a05-1	1,00	66	0,753	2.333	2.464	0,95
P3a05-2	1,00	66	0,753	2.454	2.464	1,00
P4a15-1	0,75	76	0,732	2.584	2.704	0,96
P4a15-2	0,75	76	0,732	2.609	2.704	0,97
P4a10-1	0,75	82	0,722	2.603	2.851	0,91
P4a10-2	0,75	82	0,722	2.598	2.851	0,91
P4a05-1	0,75	72	0,739	2.222	2.606	0,85
P4a05-2	0,75	72	0,739	2.199	2.606	0,84
P3p10-2	0,50	54	0,785	2.391	2.197	1,09

Figure 26 – Failure of columns test in the second stage

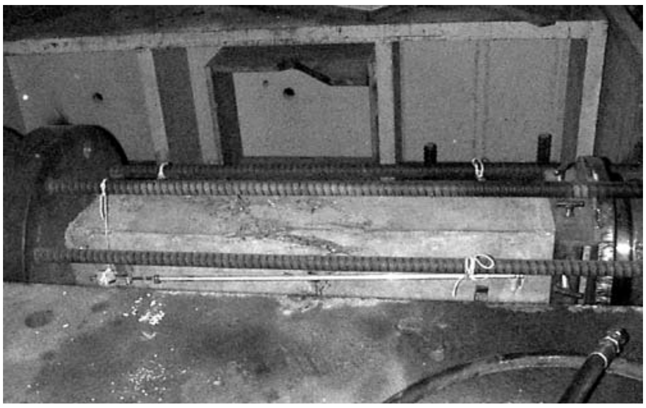


Table 3 – Columns Tests Summary

Column	Vf	$0,9_{fcm}$ (MPa)	Stirrups	Acn (cm^2)	$F_{u,exp}$ (kN)	$F_{u,teo}$ (kN)	$F_{u,exp}/F_{u,teo}$
P40a05	0,51%	59	∅6,3c/05	103,45	2.384	1.114	2,14
P40a10	0,51%	59	∅6,3c/10	103,45	2.022	1.114	1,82
P40a15	0,51%	59	∅6,3c/15	103,45	2.244	1.114	2,01
P60a05	0,76%	64	∅6,3c/05	103,45	2.429	1.154	2,11
P60a10	0,76%	64	∅6,3c/10	103,45	2.121	1.154	1,84
P60a15	0,76%	64	∅6,3c/15	103,45	2.007	1.154	1,74
P80a05	1,02%	68	∅6,3c/05	103,45	1.830	1.199	1,53
P80a10	1,02%	68	∅6,3c/10	103,45	2.184	1.199	1,82
P80a15	1,02%	68	∅6,3c/15	103,45	1.973	1.199	1,65

5 Conclusions

- It was noted in the two stages of the test that the cover doesn't detach from the column before its collapse. The fibres serve as a link to avoid the rupture of the cover with a smaller load than that of the collapse;
- Only the column core, delimited by the stirrups, formed the cross section resistant to normal compression stress. In high strength steel-fibre-reinforced concrete columns the conclusion was not different. This means that only the core cross section contributed to absorb the applied load. This took place in columns with a 2cm concrete cover thickness;
- when the concrete cover thickness was increased to 5cm it could be noted that the core did not only contribute to the strength of the column cross section, but a part of the cover also contributed to the ultimate strength of the column;
- There wasn't any detachment of the cover before the rupture of the column, which confirms the idea about the cover participation to the section strength. Therefore, the use of thicker concrete cover in the high strength concrete columns construction with steel-fibre reinforcement is indicated.

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